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Design Aids for Stiffened Channels

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Introduction

The cold formed channel is a valuable member of the family of steel sections. Its greatest advantage is economy -- which can be attributed to this section's highly favourable strength to weight ratio. The cold formed channel is widely used in construction and manufacturing. However, usage is far less than it should be for three reasons:

1. Absence of standard sizes.
2. Inconsistent properties and strengths in design aids.
3. Unavailability of small quantities.

The most important reason cold formed channels have not reached their market potential is the absence of standard sizes. Designers are reluctant to specify a proprietary product or one that is only available from a single source. Of interest, the Association of Steel Manufacturers realized this fact was detrimental to the sale of "I" beams. Thus, they collaborated and published industry standard sizes for "I" beams in 1896! While cold formed channels have been produced for at least fifty years, standard sizes have yet to be promulgated. The Canadian Standards Association is now addressing the issue. Their intent is to include an optimized series of standard cold formed angles and channels in CSA Standard G312.3.[1].

The fact that properties and strengths of the exact same cold formed channel differ from one handbook to another and from one manufacturer's literature to another's, is also a significant deterrent to the use of cold formed channels. This inconsistency is confusing and gives engineers a lack of confidence in the product. Referring again to "I" beams, every engineer is fully familiar with the "Bibles" of steel construction - the AISC[2] and the CISC[3] steel construction handbooks. The AISC and CSIC have arrived at standardized corner fillets, steel density, etc. which allow anyone to calculate "I" beam properties and get the exact same answer as those published in the AISC and CISC handbooks. The authority of these handbooks is such that designers readily accept their published data. We are sure that the AISC and CISC would include cold formed channels in their handbooks if standard sizes and a universal method for calculating properties and strengths were developed.

The difficulty in obtaining small quantities of cold formed channels is a direct result of the absence of standard sizes. Standard sizes permit service centres to stock steel sections. The construction industry comprises a large number of small projects. Each project usually contains a relatively small quantity of a given steel section, hence the sections are purchased from service centres. Because cold formed channels are not available in small quantities from the service centres, they are not as widely used in construction as they otherwise would be.

Objectives

The development of a standard series of cold formed channels and increasing their availability at service centres is a long process. Many organizations would be involved. However, the engineering fraternity can take the initiative to develop methods for arriving at consistent section properties and strengths for cold formed channels. The AISI[4] and CSA[5] codes for the design of cold formed members would be invaluable in this regard. Nevertheless, a number of assumptions must be made when applying these codes. For example, a user must define the flange and the web in a cold formed channel. Specifically, are the corner radii a part of the web or a part of the flange? Due to the necessity to answer questions such as this one, the properties and strengths of the exact same cold formed channel differ from one handbook to another and from one manufacturer's literature to that of another. The authors' wish is that this paper will lead to the standardization of engineering calculations for cold formed stiffened channels. To this end, a complete set of calculations for the section properties and beam and column strengths of an example cold formed stiffened channel is given. Computer-generated beam and column tables are also given for the same section.

Example Calculations and Tables

1.0 Channel section

The example calculations will be based on the typical channel with stiffened flanges shown in Figure 1. The channel is made from galvanized steel (A446M Grade D) and has a Z275 coating weight (ASTM A525M). The thickness of the steel core, excluding any coating, is 2.5 mm. The yield strength of A446M Grade D steel is 345 MPa and the tensile strength is 450 MPa. Most calculations will be performed to three significant figures for clarity of presentation. However, intermediate computer calculations are not rounded off and the computer tables may differ slightly from the "hand" calculations shown. SI units and working stress design in accordance with CSA Standard S136 - 1974 "Cold Formed Steel Structural Members"[5] revised to April 1982 will be used.

We will determine:

(a) Applicable code restrictions.
(b) Required section properties.
(c) Beam strength about both major axes.
(d) Column strength under concentric loading.

2.0 Check section for code restrictions for bending in the direction of major (X-X) axis.

2.1 Check flange (Section 4.3.1 (a)(i))
The maximum flat width ratio W (= w/t) for a stiffened compression element having one longitudinal edge connected to a web or flange element, the other stiffened by a simple lip bent at right angles to the element, is $W \leq 60$.

$$W = \frac{w}{t} = \frac{75}{2.5} = 30$$

∴ no restriction applies.

2.2 Check web (Clause 4.3.2)
Maximum allowable web depth is restricted to an H $(= \frac{h}{t})$ ratio of:
(a) 150 for members subject to web crippling and
(b) 200 for other members.

$$H = \frac{h}{t} = \frac{245}{2.5} = 98$$

∴ our section satisfies both conditions.

2.3 Check edge stiffener (Clause 4.6.1)
A flat compression element may be considered as a stiffened compression element, if it is stiffened along one longitudinal edge by a web, and along the other edge by a lip whose overall depth is:

$$d_l \geq t(24W - 156)^{1/3} \text{ and } 4.8\,t$$

$$2.5\,(24 \times \frac{75}{2.5} - 156)^{1/3} = 20.7 \text{ mm}$$

$$4.8 \times 2.5 = 12 \text{ mm}$$

$$\therefore d_l = 25 \text{ mm satisfies clause 4.6.1.}$$

2.4 Calculate increase in yield strength due to cold work.
We wish to stress the section to its full allowable stress,

i.e. $f = qF = \frac{qFy}{1.60}$ or $f = qF = \frac{qF'y}{1.60}$ (Clause 5.3).

In a stiffened channel, the flange consists of the flat portion plus the two 90° corners (e.g. elements 2, 3 and 4 in Figure 1).

Assume the compression flange is fully effective. Then, qf can equal $\frac{qF'y}{1.60}$ (Clause 3.3.1).

$F'y = Fy + 5D(Fu - Fy)/W^*$ (Clause 3.3.1)

$Fy = 345$ MPa, $Fu = 450$ MPa

D = no. of 90° corners = 2

$$W^* = \frac{\text{flange centreline length}}{t} = \frac{75 + \pi\left(\frac{5 + 7.5}{2}\right)}{2.5} = 37.9$$

$F'y = 345 + 5(2)(450-345)/37.9 = 373$ MPa

q = 1.0 for a stiffened element (Clause 4.8 a))

$$\therefore f = \frac{qF'y}{1.60} = \frac{1.0 \times 373}{1.60} = 233 \text{ MPa}$$

2.5 Calculate effective width of compression flange.
Determine effective width from Clause 4.5.2.1.

$B \le W$ and B_{FS}

$W = 30$

Clause 4.5.2.1 a)

$B_{FS} = 1.30\, g - R$

$$g = \sqrt{\frac{E}{f}}$$

E = 203 000 MPa (Clause 13.4)

$$g = \sqrt{\frac{203\,000}{233}} = 29.5$$

$R = 0$ since $W < 60$

$B_{FS} = 1.30\,(29.5) - 0 = 38.4$

$\therefore B = W = 30$

$\therefore$ Our assumption that the compression flange is fully effective under the allowable stress of 233 MPa is correct. (Note: Section 9.2 illustrates the case when the compression flange is not fully effective).

3.0 Determine section properties about the major (X - X) axis for the gross area. See Table 1.

4.0 Moment of inertia about the major (X - X) axis for deflection calculations. Clause 4.5.2.1.

Since our compression flange is fully effective, the moment of inertia for deflection calculations is the same as for load calculations. (Note: Section 9.2 illustrates the case when the compression flange is not fully effective).

5.0 Check whether compressive stress in the web or flange governs.

In Section 2.4, the allowable stress in the flange (Point B, Figure 1) was established as 233 MPa.

The allowable stress in the flat web (i.e. Point A, Figure 1) according to Clause 5.5.2 is:

$F_w \leq F$

$\leq$ the greater of $17.6\, E/H^2$ and $(1.21 - 0.058\, H/g_y)F$

The utilization of cold work of forming does not apply to allowable stresses in the web (Clause 3.3).

$\therefore F = \dfrac{Fy}{1.60}$ (Clause 5.2) $= \dfrac{345}{1.60} = 216$ MPa

$17.6\, E/H^2 = 17.6 \times 203\,000/98^2 = 372$ MPa

$(1.21 - 0.058\, H/g_y)F$

$g_y = \sqrt{E/Fy} = \sqrt{203\,000/345} = 24.3$

$(1.21 - 0.058 \times 98/24.3)\, 216 = 211$ MPa

F_w must be less than or equal to a) F = 216 and b) the greater of 372 and 211.

$\therefore F_w = 216$ MPa

If the bending stress at Point A is not to exceed F_w then the stress at Point B must be limited to (see Figure 1):

$F_w \times \dfrac{A'}{a} = 216 \times \dfrac{250}{235} = 230$ MPa

Thus, the prorated stress of 230 MPa is less than the allowable stress in the flange (233 MPa at Point B).

$\therefore$ the governing stress is 230 MPa at Point B.

6.0 Calculate allowable loading for bending about major axis

6.1 Calculate section modulus for bending about the X - X axis.

$$S_x = \frac{Ix\text{-}x\ (C\,G)}{\bar{y}} = \frac{10\,920 \times 10^3}{125} = 87\,400 \text{ mm}^3$$

6.2 Calculate allowable bending moment

M = S_x x allowable stress = 87 400 x 230/1000 = 20 100 N.m

6.3 Calculate allowable web shear stress

Clause 5.5.1 stipulates the maximum average shear stress on the gross area of a flat web shall not exceed Fy/2.50, nor

(a) when $H \leq 3.2\, g_y$, $0.88E/H\, g_y$

(b) when $H > 3.2\, g_y$, $2.82E/H^2$

$F_y/2.5 = 345/2.5 = 138$ MPa

H = 98

$3.2\, g_y = 3.2\sqrt{E/F_y} = 3.2\sqrt{203\,000/345} = 77.6$

$\therefore H > 3.2\, g_y$ and $2.82\, E/H^2 = 2.82 \times 203\,000/98^2 = 59.6$ MPa

$\therefore F_v = 59.6$ MPa

6.4 Calculate allowable web shear force

V = F_v x gross area of flat web

= 59.6 x 235 x 2.5 = 35 000 N

6.5 Calculate allowable loading

Two conditions must be **satisfied:**

(a) Maximum moment $\leq M$
(b) Total load (W) $\leq 2V$

Assuming uniformly distributed load on a simply supported span:

(a) $M_{max} = M = \frac{W L}{8}$

and $W \leq \frac{M \times 8}{L}$

(b) $W \leq 2V \leq 2 \times 35\,000 \leq 70\,000$ N

For a 6 m span:

$W \leq \frac{20\ 100 \times 8}{6} = 26\,800$ N

$\therefore$ M governs and W = 26 800 N

For a 2 m span:

$W \leq \frac{20\ 100 \times 8}{2} = 80\,400$ N

$\therefore$ shear governs and W = 70 000 N

Allowable loads for a given section, along with section properties, are usually summarized in computer generated tables. One such example is shown in Table 2.

7.0 Web crippling for loading about major axis (Clause 5.5.4).

7.1 End reaction in bearing

H = 98 is less than the upper limit of 150 on H given in Clauses 4.3.2 and 5.5.4.

Inside corner radius = 5.0 is less than 4t = 4 x 2.5 = 10.

Therefore, in order to avoid crippling of the unreinforced beam web, the maximum end reaction permitted is:

$P_{max.} = 0.01\ t^2 F_y (98 + 4.20\ N - 0.022NH - 0.011\ H)(1.15 - 0.15n)(4-k)$

$t = 2.5$ mm, $F_y = 345$ MPa

$n = \frac{\text{inside bend radius}}{t} = \frac{5}{2.5} = 2$

$k = (29.9/g_y)^2 = \left(29.9/\sqrt{203\,000/345}\right)^2 = 1.52$

If we set $N = \frac{\text{bearing length}}{t}$ equal to the maximum allowable (H = 98),

$P_{max} = 0.01 \times 2.5^2 \times 345\ (98 + 4.20 \times 98 - 0.022 \times 98 \times 98 - 0.011 \times 98)$

$(1.15 - 0.15 \times 2)\ (4 - 1.52) = 13\,500$ N

Thus, for a bearing length of Nt = Ht = 98 x 2.5 = 245 mm, the maximum end reaction permitted is 13 500 N. Beams having larger end reactions or a shorter bearing length must have reaction stiffeners or the end reaction must be resisted by an end shear, rather than an end bearing, connection.

7.2 Concentrated loads on span

Assuming that the concentrated load is applied over the maximum permissible bearing length (Nt = Ht = 245 mm),

$P_{max} = 0.01 t^2 F_y (305 + 2.30N - 0.009NH - 0.5H)(1.06 - 0.06n)(3.67 - 0.67k)$

$= 0.01\ (2.5)^2 (345)(305 + 2.30 \times 98 - 0.009 \times 98 \times 98 - 0.5 \times 98)\ (1.06 - 0.06 \times 2)$

$(3.67 - 0.67 \times 1.52) = 21\,200$ N

Therefore, for a bearing length of Nt = Ht = 245 mm, the maximum concentrated load permitted on the span is 21 200 N. For larger concentrated loads or a shorter bearing length, web stiffening would be required.

8.0 Determine unbraced length
The maximum unbraced length permitted by S136 when the channel is loaded as a beam about the "X-X" axis (under loading which causes the allowable stress to be reached) will be calculated.

Clause 5.4.2.1 applies. Clause 3.3 permits the utilization of cold work of forming when applying 5.4.2.1. Therefore, the allowable stresses are those which were previously calculated. The allowable compressive stress on the extreme fibre (F_c) is 230 MPa (see section 5.0). Using Clause 5.4.2.1, this value of F_c will be used to calculate the maximum permitted unbraced length.

i) $F' = 1.11\ F$
Since $F = F_c = 230$ MPa,
$F' = 1.11 \times 230 = 255$ MPa

ii) $F_t = 0.174\ GAt^2\ C_b/dS_{xc}$
$G = 77\ 900$ MPa (Clause 13.4)
$A = 1\ 148.3\ mm^2$, $t = 2.5$ mm

C_b can be conservatively taken as 1.0. If we take $C_b = 1.0$, then our solution can be applied to any condition of beam curvature. Also, for a simply supported, unbraced beam, $C_b = 1.0$. Take $C_b = 1.0$, d = 250 mm

$$S_{xc} = \frac{I_{x-x}}{d/2} = \frac{10\ 920 \times 10^3}{250/2} = 87\ 400\ mm^3$$

$$F_t = 0.174 \times 77\ 900 \times 1\ 148.3 \times 2.5^2 \times 1.0/250 \times 87\ 400 = 4.45\ MPa$$

iii) When $F_{be} > 0.5\ (F' - F_t)$, $F_c \le F' - \left[\frac{0.25(F' - F_t)^2}{F_{be}}\right]$ and F

When $F_{be} \le 0.5\ (F' - F_t)$, $F_c = F_{be} + F_t$

Assume $F_{be} > 0.5\ (F' - F_t)$. Then,

$$F_c = 230 = 255 - \frac{0.25\ (255 - 4.45)^2}{F_{be}}$$

$F_{be} = 628$ MPa

$0.5\ (F' - F_t) = 0.5\ (255 - 4.45) = 125$ MPa

$\therefore F_{be} > 0.5\ (F' - F_t)$ as assumed and $F_{be} = 628$ MPa.

iv) $F_{be} = 5.12\ Ed\ I_{yc}\ C_b/L^2\ S_{xc}$ or $L = \sqrt{\frac{5.12\ Ed\ I_{yc} C_b}{S_{xc}}}$

$E = 203\ 000$ MPa (Clause 13.4)

$I_{yc} = \frac{I_{Y-Y}}{2}$ since the channel is symmetrical and fully effective.

$I_{Y-Y} = 1\ 220 \times 103$ (see Table 3)

$$I_{yc} = \frac{1\ 220 \times 10^3}{2} = 610 \times 10^3\ mm^4$$

$$L = \sqrt{\frac{5.12(203\ 000)(250)(610 \times 10^3)(1.0)}{87\ 400}} = 1700\ mm$$

9.0 Bending About Minor (Y-Y) Axis

There are two situations to be considered:

(a)  and  (b)

9.1 Consider situation (a)

9.1.1 Check maximum allowable web depth (Clause 4.3.2). In this mode of loading, elements 3 and 7 become webs. Since $H = \frac{85}{2.5} = 34$ there is no restriction.

9.1.2 Check the flat width ratios of the compression elements. The lips are unstiffened compression elements and Clause 4.3.1(c) restricts their flat width ratio to 60.

$$W = \frac{w}{t} = \frac{17.5}{2.5} = 7 < 60 \quad \text{Therefore OK.}$$

9.1.3 Calculate allowable stresses.

Section 3.3 permits the utilization of cold work of forming for the tension flange. Therefore,

$$F_t = \frac{q\,F'_y}{1.60} \qquad \text{(Clause 5.3)}$$

$$q = 1.0 \qquad \text{(Clause 4.8a)}$$

$$F'_y = F_y + 5D(F_u - F_y)/W^*$$

$$= 345 + 5(2)(450 - 345) \Big/ \frac{235 + \pi\frac{(5 + 7.5)}{2}}{2.5} = 355\ MPa$$

$$F_t = \frac{1.0 \times 355}{1.60} = 222\ MPa$$

Section 3.3 permits the utilization of cold work of forming for the compression flange if it is not subject to local buckling. The compression flange consists of the two lips and the two 90° corners connecting the lips to the webs. Assume elements 1 and 9 are not subjected to local buckling.

$$F'_y = F_y + 5D\,(F_u - F_y)/W^* \quad \text{(Clause 3.3.1)}$$

$$= 345 + 5(2)(450 - 345) \Big/ \frac{(2 \times 17.5) + \pi\frac{(5 + 7.5)}{2}}{2.5} = 393\ MPa$$

$$g'_y = \sqrt{E/F'_y} \text{(Clause 2.2)} = \sqrt{203\ 000/393} = 22.7$$

$0.37\,g'_y = 0.37 \times 22.7 = 8.4$ (Clause 4.8b) i))

$\therefore$ Since $W = 7 < 0.37\,g'_y$, elements 1 and 9 are not subjected to local buckling and cold work of forming may be utilized.

$$q = 1.0, \qquad \text{(Clause 4.8b) i))}$$

$$F = \frac{q\,F'_y}{1.60} \qquad \text{(Clause 5.3)}$$

$$= \frac{1.0 \times 393}{1.60} = 246\ MPa$$

The allowable stress at Point D (see Fig. 1) is 246 MPa.

The stress at Point C (see Fig. 1) cannot exceed F_w. Clause 5.5.2 gives:

$F_w \leq$ a) F and b) the greater of $17.6\ E/H^2$ and $(1.26 - 0.087\ H/g_y)F$

$F = \frac{345}{1.60} = 216$ MPa

$17.6\ E/H^2 = 17.6 \times 203\ 000/34^2 = 3090$ MPa

$(1.26 - 0.087\ H/g_y)F = (1.26 - 0.087 \times 34/24.3)\ 216 = 246$

$\therefore F_W \leq 216$

$\leq$ the greater of 3090 and 246

and $F_W = 216$ MPa

If the stress at Point C is 216 MPa, then the stress at Point D would be:

$216 \times \frac{64}{56.5} = 245$ MPa

$\therefore$ the governing compressive stress (F_c) at Point D is 245 MPa.

9.1.4 Determine section properties about the minor (Y - Y) axis for the gross area. See Table 3.

9.1.5 Resisting moment for "Y-Y" axis bending (case a)).

$S_t = \frac{I_{y-y}}{x_t} = \frac{1\ 220 \times 10^3}{26} = 46\ 900$ mm^3

$S_c = \frac{I_{y-y}}{x_c} = \frac{1\ 200 \times 10^3}{90 - 26} = 19\ 100$ mm^3

$M_t = S_t \times F_t = 46\ 900 \times 222/1\ 000 = 10\ 400$ N.m

$M_c = S_c \times F_c = 19\ 100 \times 245/1\ 000 = 4\ 680$ N.m

$\therefore M = 4\ 680$ N.m

Note: It is assumed that the unbraced compression flanges will not buckle laterally.

9.2 Consider bending about the Y-Y axis for situation (b).

9.2.1 Determine effective width of compression flange. Element 5 (see Figure 1) is in compression and must be checked to see if it is fully effective.

The allowable stress in the tension flange is

$F_t = \frac{q\ F'_y}{1.60}$ (Clauses 3.3 and 5.3)

F'_y was previously calculated as 393 MPa for the lips and adjoining corners.

$q = 1.0$ (Clause 4.8b)

$F_t = \frac{1.0 \times 393}{1.60} = 246$ MPa

It is obvious that the neutral axis will be closer to the compression flange than the tension flange. $\therefore$ The maximum allowable tensile stress will govern the design.

The compressive stress, when the maximum tensile stress is reached, will be:

$$F_t \times \frac{x_c}{x_t}$$

Assume $x_c = 30$

$\therefore f_c = 246 \times \frac{30}{90 - 30} = 123$ MPa

Clause 4.5.2.1a)

$B \leq W$

$\leq B_{FS} \leq 1.30\,g - R$

$W = \frac{235}{2.5} = 94$ O'K, since less than 500 (Clause 4.3.1 b))

$g = \sqrt{E/f_c} = \sqrt{203\,000/123} = 40.7$

$R = 0$ since element 5 is stiffened at each edge by a corner and web.

$B_{FS} = 1.30(40.7) = 53$

$\therefore B = 53$, $b = 53t = 53 \times 2.5 = 132.5$ mm $\therefore$ Element 5 is not fully effective and its area is reduced as shown in Figure 2

Element 10 (see Figure 2)

$A = -102.5 \times 2.5 = -256.3\ \text{mm}^2$

$Ax = -256.3 \times \frac{2.5}{2} = -320\ \text{mm}^3$

To obtain effective area and the neutral axis of the effective section, deduct element 10 properties from the gross section properties in Table 3.

$\Sigma A_{eff} = 1\,148.3 - 256.3 = 892\ \text{mm}^2$

$\Sigma Ax_{eff} = 29\,790 - 320 = 29\,470\ \text{mm}^3$

$\bar{x}_{eff} = \frac{29\,470}{892} = 33$ mm

Therefore, assumed $x_c = 30$ is not correct.

Assume $x_c = 34$

$f_c = 246 \times \frac{34}{90 - 34} = 149$ MPa

$g = \sqrt{203\,000/149} = 36.9$

$B = 1.30 \times 36.9 = 48$

Element 10,

$A = -(235 - 48 \times 2.5)2.5 = -287.5$

$Ax = -287.5 \times 1.25 = -360$

$\Sigma A_{eff} = 1\,148.3 - 287.5 = 860.8$

$\Sigma Ax_{eff} = 29\,790 - 360 = 29\,430$

$\bar{x}_{eff} = \frac{29\,430}{860.8} = 34$ mm

Therefore, assumed $x_c = \bar{x}_{eff}$ and the effective width of the compression flange is $48t = 48(2.5) = 120$ mm.

9.2.2 Determine properties of effective section.
Element 10 properties will be deducted from the gross section properties in Table 3.

Element 10,

$Ax^2 = -287.5 \times 1.25^2 = \text{negligible}$

$I_o = -\dfrac{115 \times 2.5^3}{12} = \text{negligible}$

ΣAx^2 and ΣI_o remain unchanged at 1820×10^3 and 180×10^3 respectively.

$I_{y-y}(CGeff) = 1\,820 \times 10^3 + 180 \times 10^3 - 860.8 \times 34^2 = 1\,000 \times 10^3$

$S_{t_{eff}} = \dfrac{I_{y-y}(C\,Geff)}{x_t} = \dfrac{1\,000 \times 10^3}{90 - 34} = 17\,900$

9.2.3 Determine resisting moment

$M_{eff} = S_{t_{eff}} \times F_t = 17\,900 \times 246/1\,000 = 4\,400$ N.m

9.2.4 Deflection determination. The effective width is determined from Clause 4.5.2.1 c). The moment of inertia for deflection determination is calculated using this effective width.

9.3 As shown in sections 6, 7 and 8, allowable loading, end reactions, concentrated loads and span can be calculated. See Tables 4 and 5.

10.0 Column design.

Determine the allowable loading in axial compression for the section of Figure 1. Assume the ends of the member are free to rotate about both major axes and the unbraced length about both major axes is 3 m.

10.1 Determine section properties:

- A - gross cross-sectional area,
- r_x, r_y - radii of gyration of gross cross-sectional area about principal axes,
- x_o - distance along X axis from shear centre to centre of gravity,
- J - St. Venant torsion constant for thin walled sections,
- C_w - warping constant of torsion of the cross-section.

10.2 A, r_x, r_y.

From Table 1, $A = 1\,148.3\ \text{mm}^2$

$I_{x-x}(CG) = 10\,920 \times 10^3\ \text{mm}^4$

$$\therefore r_x = \sqrt{\frac{10\,920 \times 10^3}{1\,148.3}} = 97.5\ \text{mm}$$

From Table 3, $I_{Y-Y}(CG) = 1\,220 \times 10^3\ \text{mm}^4$

$$\therefore r_y = \sqrt{\frac{1\,220 \times 10^3}{1\,148.3}} = 32.6\ \text{mm}$$

10.3 x_o, J, C_w

Formulae for these properties are given on pages III 16 and III 17 of the AISI "Cold-Formed Steel Design Manual"[6].

See Figure 1 for nomenclature.

$$m = \frac{\bar{b}t}{12I_x}\left[6\bar{c}(\bar{a})^2 + 3\bar{b}(\bar{a})^2 - 8(\bar{c})^3\right]$$

$$= \frac{87.5(2.5)}{12(10\ 920 \times 10^3)}\left[6(23.75)(247.5)^2 + 3(87.5)(247.5)^2 - 8(23.75)^3\right]$$

$$= 41.2 \text{ mm}$$

From Table 3, $\bar{x}$ = 26 mm

Note: The AISI measures $\bar{x}$ to the centre of the web. Therefore, a $\frac{t}{2}$ adjustment $\bar{x}$ as calculated in this paper is necessary.

$$x_o = m + \bar{x} - \frac{t}{2} = 41.2 + 26 - \frac{2.5}{2} = 66.0 \text{ mm}$$

$$J = \frac{t^3}{3}(a + 2b + 4u + 2c) = \frac{2.5^3}{3}(235 + 2(75) + 4(9.82) + 2(17.5))$$

$$= 2\ 390 \text{ mm}^4$$

$$C_w = \frac{t^2}{A}\left\{\frac{\left(\bar{x} - \frac{t}{2}\right) A\,(\bar{a})^2}{t}\left[\frac{(\bar{b})^2}{3} + m^2 - m\,\bar{b}\right]\right.$$

$$+ \frac{A}{3t}\left[(m)^2\,(\bar{a})^3 + (\bar{b})^2\,(\bar{c})^2\,(2\bar{c} + 3\bar{a})\right]$$

$$- \frac{I_x\,m^2}{t}(2\bar{a} + 4\bar{c}) + \frac{m(\bar{c})^2}{3}\left[8(\bar{b})^2\bar{c} + 2\,m\,\left(2\bar{c}(\bar{c} - \bar{a}) + \bar{b}(2\bar{c} - 3\bar{a})\right)\right]$$

$$\left. + \frac{(\bar{b})^2\,(\bar{a})^2}{6}\left[(3\bar{c} + \bar{b})(4\bar{c} + \bar{a}) - 6(\bar{c})^2\right] - \frac{m^2\,(\bar{a})^4}{4}\right\}$$

$$= \frac{2.5^2}{1\ 148.3}\left\{\frac{\left(26 - \frac{2.5}{2}\right)(1\ 148.3)(247.5)^2}{2.5}\left[\frac{87.5}{3}^2 + 41.2^2 - (41.2)(87.5)\right]\right.$$

$$+ \frac{1\ 148.3}{3(2.5)}\left[(41.2^2)(247.5)^3 + (87.5^2)(23.75^2)(2 \times 23.75 + 3 \times 247.5)\right]$$

$$- \frac{10\ 920\ (10^3)(41.2^2)}{2.5}(2 \times 247.5 + 4 \times 23.75)$$

$$+ \frac{(41.2)(23.75^2)}{3}\left[8(87.5^2)(23.75)\right.$$

$$\left. + 2(41.2)\left((2)(23.75)(23.75 - 247.5) + (87.5)(2 \times 23.75 - 3 \times 247.5)\right)\right.$$

$$+ \frac{(87.5^2)(247.5^2)}{6}\left[(3 \times 23.75 + 87.5)(4 \times 23.75 + 247.5) - 6(23.75^2)\right]$$

$$- \frac{(41.2^2)(247.5)^4}{4}\Bigg\} = 1.58 \times 10^{10} \text{ mm}^6$$

10.4 Utilization of cold work of forming.
Assume Q (Clause 4.9) is less than 1.0
$\therefore$ Cold work of forming cannot be utilized and F_y = 345 MPa (Clause 3.3.2a))

10.5 Determine F_p (reduced elastic buckling stress). Clause 5.6.3 applies since we have a singly symmetric shape.

F_e (Euler elastic buckling stress) is determined from Clause 5.6.2.

$$F_e = \frac{5.12\ E}{Y^2}$$

E = 203 000 MPa (Clause 13.4)
Y = greater of the effective slenderness ratios about X or Y axes.

The unbraced member length (L) about both axes is 3 000 mm (see section 10.0). Both ends of the member are free to rotate about both the X and Y axes. $\therefore$ the effective length factor (K) is 1.0 about both the X and Y axes.

$$\frac{KL}{r_x} = \frac{1.0\ (3\ 000)}{97.5} = 30.8$$

$$\frac{KL}{r_y} = \frac{1.0\ (3\ 000)}{32.6} = 92.0$$

$$\therefore Y = 92.0$$

$$\therefore F_e = \frac{5.12\ (203\ 000)}{(92.0)^2} = 123 \text{ MPa}$$

F_{st} (Torsional - flexural elastic buckling stress) is determined from Clause 5.6.3.

$$F_{st} = \frac{1}{2\beta}\left[F_s + F_t - \sqrt{(F_s + F_t)^2 - 4\beta\ F_s F_t}\ \right]$$

F_s = Euler elastic buckling stress about the axis of symmetry

$$= \frac{5.12\ E}{S^2} = \frac{5.12\ E}{(KL/r_x)^2} = \frac{5.12\ (203\ 000)}{30.8^2} = 1\ 096 \text{ MPa}$$

$$F_t = \text{Torsional elastic buckling stress} = \frac{0.52}{Ar_o^2}\left[G J + \frac{\pi^2 E}{(KL)^2} C_w\right]$$

G = 77 900 MPa (Clause 13.4)

$$r_o^2 = r_x^2 + r_y^2 + x_o^2 = 97.5^2 + 32.6^2 + 66.0^2 = 14\ 900 \text{ mm}^2$$

$$F_t = \frac{0.52}{1148.3(14\ 900)}\left[77\ 900(2390) + \frac{\pi^2(203\ 000)}{(1.0 \times 3000)^2}(1.58)(10^{10})\right] = 113 \text{ MPa}$$

$$\beta = 1 - (x_o/r_o)^2 = 1 - x_o^2/r_o^2 = 1 - (66.0^2/14\ 900) = 0.708$$

$$F_{st} = \frac{1}{2(0.708)}\left[1096 + 113 - \sqrt{(1096 + 113)^2 - 4(0.708)(1096)(113)}\right]$$

$= 109$ M Pa

$F_p = F_{st}$ or F_e, whichever is less.

$F_p = 109$ M Pa

10.6 Determine allowable axial stress (QF_a).

i) Determine F_a

$F = \frac{F_y}{1.60}$ (Clause 5.2 & section 10.4)

$F = \frac{345}{1.60} = 216$ M Pa

Clause 5.6.1a), when $F_p > F/2$, $F_a = F - \left[F^2/4\ F_p\right]$,

when $F_p \leq F/2$, $F_a = F_p$

$\frac{F}{2} = 108$ M Pa

$F_p = 109 > \frac{F}{2}$

$\therefore F_a = 216 - \left[216^2/(4)\ (109)\right] = 109$ M Pa

ii) Determine Q

$Q = Q_f\ Q_a$ (Clause 4.9)

Q_f = least value of q determined from Clause 4.8 using:

(a) $F_y = 1.6\ F_a = 1.6\ (109) = 174$ MPa

(b) $g_y = \sqrt{E/1.6\ F_a} = \sqrt{203\ 000/174} = 34.2$ (Clause 5.6.1)

q = 1.0 for stiffened elements 3, 5 & 7 in Figure 1.

$W = \frac{17.5}{2.5} = 7$ for unstiffened elements 1 & 9 in Figure 1.

$0.37\ g_y = 0.37\ (34.2) = 12.7$

$\therefore W < 0.37\ g_y$ & q = 1.0 for elements 1 & 9.

$\therefore q_{min}$ = least value of q

$\therefore Q_f = q_{min} = 1.0$

$Q_a = (\Sigma B_f t^2 + \Sigma A_c)/A$ (Clause 4.9).

For elements 1 & 9, $B_f = W = 7$

For elements 3, 7 & 5:

$B_f \leq \left(1.64\ g_y/\sqrt{Q_f}\right) - R$ and W

R = 0 for element 5 since it is stiffened at each edge by a flange (Clause 4.5.2.1).

R = 0 for elements 3 & 7 since $W = \frac{75}{2.5} = 30 < 60$.

For elements 3 & 7,

$\left(1.64\ (34.2)/\sqrt{1}\right) - 0 = 56$

$\therefore B_f = 30$

For element 5,

$\left(1.64\,(34.2)/\sqrt{1}\right) - 0 = 56.\quad \frac{235}{2.5} = 94$

$\therefore B_f = 56$

$\Sigma A_c = 98.2,\ A = 1148.3$ (see Table 1)

$$Q_a = \frac{2(7)2.5^2 + 2(30)2.5^2 + (56)2.5^2 + 98.2}{1148.3} = 0.793$$

$Q = Q_f\,Q_a = 1.0\,(0.793) = 0.793$

iii) Determine QF_a

$\therefore$ The assumption that $Q < 1.0$ in section 10.4 is correct.

$\therefore$ Actual $QF_a = 0.793\,(109) = 86$ MPa

10.7 Determine allowable load in axial compression.

Allowable axial load $= QF_aA$

$= 86\,(1148.3)$

$= 98\,800$ N

10.8 An example column design aid is shown in Table 6.

Conclusions

The use of cold formed channels is hindered by the absence of standard sizes, inconsistent properties and strengths in design aids, and the unavailability of small quantities at service centres. Standard sizes and unavailability are outside the scope of this paper. However, this paper sets forth precise methods for calculating the section properties and the beam and column strengths of cold formed channels. The authors' wish is that this paper will lead the engineering fraternity to the establishment of a common approach for cold formed channel calculations. Their ultimate hope is that North American handbooks and design aids will provide consistent properties and strengths for cold formed channels.

Appendix - References

1. Canadian Standards Association, "Preferred Metric Dimensions for Structural Steel W and HP Shapes, Angles, and Hollow Structural Sections, CAN3-G312.3-M78", Rexdale, Ontario, 1978.

2. American Institute of Steel Construction, Inc., "Manual of Steel Construction", Seventh Edition, New York, 1970.

3. Canadian Institute of Steel Construction, "Handbook of Steel Construction", Third Edition, Willowdale, Ontario, 1980.

4. American Iron and Steel Institute, "Specification for the Design of Cold-Formed Steel Structural Members", Washington, D.C., May 1983.

5. Canadian Standards Association, "Cold Formed Steel Structural Members, CSA Standard S136-1974", Rexdale, Ontario, 1974.

6. American Iron and Steel Institute, "Cold Formed Steel Design Manual", Washington, D.C., May 1983.

Appendix - Notations

A	= area of gross section (mm^2)
A_{eff}	= area of effective section (mm^2)
b	= effective design width (mm)
B	= effective width ratio (b/t)
B_{FS}, B_L	= limiting values of B
C_b	= bending coefficient
C_w	= warping constant of torsion (mm^6)
d	= depth of section (mm)
d_l	= overall depth of lip (mm)
D	= number of 90^o corners
E	= Young's Modulus (203 000 MPa)
f	= actual stress (MPa)
f_c	= actual compressive stress (MPa)
F	= basic design stress (MPa)
F'	= 1.11F (MPa)
F_a	= allowable stress under axial compression (MPa)
F_{be}	= buckling stress of a beam (MPa)
F_c	= allowable compressive stress (MPa)
F_e	= Euler elastic buckling stress (MPa)

F_p = reduced elastic buckling stress (MPa)
F_s = Euler buckling stress about axis of symmetry (MPa)
F_{st} = torsional-flexural elastic buckling stress (MPa)
F_t = allowable tensile stress, torsional elastic buckling stress (MPa)
F_u = tensile strength of virgin steel (MPa)
F_v = allowable shear stress (MPa)
F_w = allowable compressive stress in the web (MPa)
F_y = tensile yield strength of virgin steel (MPa)
F'_y = average tensile yield strength which incorporates the effects of cold work of forming (MPa)
g = $\sqrt{E/f}$
g_y = $\sqrt{E/F_y}$
g'_y = $\sqrt{E/F'_y}$
G = shear modulus (77 900 MPa)
h = clear distance between flanges in the plane of the web (mm)
H = flat width ratio of web (h/t)
I_o = moment of inertia of an element about its centroidal axis (mm^4)
I_{x-x}, I_{y-y} = moment of inertia of gross area about X and Y axis respectively (mm^4)
I_{yc} = moment of inertia of the compression portion of a section about its gravity axis parallel to the web (mm^4)
J = St. Venant torsion constant (mm^4)
K = effective length factor
L = beam span (m), maximum unbraced length (mm)
M = allowable bending moment of the gross section (N.m)
M_c, M_t = bending moment based on allowable compressive and tensile stress respectively (N.m)
M_{eff} = allowable bending moment of the effective section (N.m)
M_{max} = maximum bending moment (N.m)
n = ratio of inside corner radius to thickness
N = ratio of bearing length to thickness
P_{max} = allowable end reaction or concentrated load (N)
q = stress factor
Q = local buckling factor
r_x, r_y = radii of gyration of gross area about the principal axes (mm)
S_c, S_t = section modulus for the outer compression and tension fibre of the gross area respectively (mm^3)
S_x, S_y = section modulus of the gross area about X and Y axis respectively (mm^3)

S_{xc}	=	I_{x-x} divided by the distance to the extreme compression fibre (mm^3)
$S_{t_{eff}}$	=	moment of inertia of the effective area divided by the distance to extreme tensio fibre (mm^3)
t	=	thickness (mm)
V	=	allowable shear force (N)
w	=	flat width (mm)
W	=	flat width ratio (w/t), total allowable uniformly distributed load (N)
W^*	=	ratio of flange centreline length to thickness
x_c, x_t	=	distance from neutral axis to outer compression and tension fibre respectively (mm)
$\bar{x}$	=	distance from outer fibre to neutral Y-Y axis of the gross area (mm)
x_{eff}	=	distance from outer fibre to neutral (Y-Y) axis of the effective area (mm)
x_o	=	distance along X axis from shear centre to centre of gravity (mm)
$\bar{y}$	=	distance from outer fibre to neutral X-X axis of the gross area (mm)
Y	=	greater of the effective slenderness ratios about X or Y axes

Tables and Figures

TABLE 1 "X-X" SECTION PROPERTIES
(Based on millimetres)

Element	A	y	Ay	Ay^2 ($\times 10^3$)	I_o
1	$17.5 \times 2.5 = 43.8$	$250 - 7.5 - \frac{17.5}{2} = 233.75$	10 200	2 390	$\frac{2.5 \times 17.5^3}{12} = -$
2 & 4	$\frac{\pi}{2} \times 7.5^2 = 88.4$	$250 - 7.5 + (0.424413 \times 7.5) = 245.68$	21 700	5 330	$0.109757 \times 7.5^4 = -$
	$-\frac{\pi}{2} \times 5^2 = -39.3$	$250 - 7.5 + (0.424413 \times 5) = 244.61$	-9 610	-2 350	$-0.109757 \times 5^4 = -$
3	$75 \times 2.5 = 187.5$	$250 - \frac{2.5}{2} = 248.75$	46 600	11 600	$\frac{75 \times 2.5^3}{12} = -$
5	$235 \times 2.5 = 587.5$	$250 \div 2 = 125.00$	73 400	9 180	$\frac{2.5 \times 235^3}{12} = 2\,700 \times 10^3$
6 & 8	(See 2 & 4) 88.4	$0.575587 \times 7.5 = 4.32$	380	-	(See 2 & 4) -
	-39.3	$2.5 + 0.575587 \times 5 = 5.38$	-210	-	-
7	(See 3) 187.5	$\frac{2.5}{2} = 1.25$	230	-	(See 3) -
9	(See 1) 43.8	$7.5 + \frac{17.5}{2} = 16.25$	710	10	(See 1) -
Σ	1148.3		143 400	26 160	$2\,700 \times 10^3$

$$\bar{y} = \frac{Ay}{A} = \frac{143\,400}{1148.3} = 125 \text{ mm}$$

$$I_{X-X}(CG) = Ay^2 + I_o - A\bar{y}^2 = 26\,160 \times 10^3 + 2\,700 \times 10^3 - 1148.3 \times 125^2$$

$$= 10\,920 \times 10^3 \text{ mm}^4$$

$$\text{Mass} = \frac{1148.3 \times 1000}{10^9} \times 7\,850 \text{ kg/m}^3 = 9.01 \text{ kg/m}$$

TABLE 2

```
*************************************************************
        METAL  SHAPES  INCORPORATED

        ***  BEAM  LOAD  TABLES -METRIC  ***

    TOTAL ALLOWABLE UNIFORMLY DISTRIBUTED LOAD
       SIMPLE SPAN, LATERALLY SUPPORTED
            WORKING STRESS DESIGN
*************************************************************
C-CHANNEL-(mm)
    90                         THICKNESS= 2.5 mm
   ****                        IR       = 5   mm
   *  * 25
250*
   *  *                        Fy= 345 MPa
   ****                        Fu= 450 MPa

*************************************************************
```

Span-L (mm)	Defl'n (mm)	Load (kN)	UDL to Deflection Limits- (kN) L/180	L/240	L/360
1000	.41	70.0	70.0	70.0	70.0
1500	1.39	70.0	70.0	70.0	70.0
2000	3.29	70.0	70.0	70.0	70.0
2500	5.89	64.2	64.2	64.2	64.2
3000	8.47	53.5	53.5	53.5	52.6
3500	11.54	45.9	45.9	45.9	38.7
4000	15.07	40.1	40.1	40.1	29.6
4500	19.07	35.7	35.7	35.1	23.4
5000	23.54	32.1	32.1	28.4	18.9
5500	28.49	29.2	29.2	23.5	15.7
6000	33.90	26.8	26.3	19.7	13.2
6500	39.79	24.7	22.4	16.8	11.2
7000	46.14	22.9	19.3	14.5	9.7
7500	52.97	21.4	16.8	12.6	8.4
8000	60.27	20.1	14.8	11.1	7.4
8500	68.03	18.9	13.1	9.8	6.6
9000	76.27	17.8	11.7	8.8	5.8
9500	84.98	16.9	10.5	7.9	5.2
10000	94.17	16.1	9.5	7.1	4.7

```
*************************************************************

    SECTION PROPERTIES AND DESIGN DATA
    **********************************

     Ix  (mm4)       10934.02 x 1000
     Sx  (mm3)          87.47 x 1000
     M   (Nm)        20065.09
     Lu  (mm)         1710.00
     V   (kN)           35.02
     Pint  (kN)          21.2 }  applies only for a
     Pext  (kN)          13.5 }  bearing length of  245.0 mm
     Min Lip (mm)       20.66
     Mass  (kg/m)       9.013

*************************************************************
```

TABLE 3 "Y-Y" SECTION PROPERTIES

(Based on millimetres)

Element	A	x	Ax	Ax^2 ($\times 10^3$)	I_o
1 & 9	$2 \times 17.5 \times 2.5 = 87.6$	$90 - \frac{2.5}{2} = 88.75$	7 770	690	$\frac{2 \times 17.5 \times 2.5^3}{12} = -$
2 & 8	$\frac{\pi}{2} \times 7.5^2 = 88.4$	$90 - 7.5 + (0.424413 \times 7.5) = 85.68$	7 570	650	$0.109757 \times 7.5^4 = -$
	$-\frac{\pi}{2} \times 5^2 = -39.3$	$90 - 7.5 + (0.424413 \times 5) = 84.62$	-3 330	-280	$-0.109757 \times 5^4 = -$
3 & 7	$2 \times 75 \times 2.5 = 375.0$	$\frac{90}{2} = 45.00$	16 880	760	$\frac{2 \times 2.5 \times 75}{12} = 180 \times 10^3$
4 & 6	(See 2 & 8) 88.4	$0.575587 \times 7.5 = 4.32$	380	-	(See 2 & 8) -
	-39.3	$2.5 + 0.575587 \times 5 = 5.38$	-210	-	-
5	$235 \times 2.5 = 587.5$	$\frac{2.5}{2} = 1.25$	730	-	$\frac{235 \times 2.5^3}{12} = -$
Σ	1148.3		29 790	1820	180×10^3

$$\bar{x} = \frac{Ax}{A} = \frac{29\,790}{1148.3} = 26 \text{ mm}$$

$$I_{Y-Y}(CG) = Ax^2 + I_o - A\bar{x}^2$$

$$= 1820 \times 10^3 + 180 \times 10^3 - 1148.3 \times 26^2$$

$$= 1220 \times 10^3 \text{ mm}^4$$

TABLE 4

```
***********************************************************************
        METAL   SHAPES   INCORPORATED

          *** BEAM  LOAD   TABLES-METRIC ***

     TOTAL ALLOWABLE  UNIFORMLY DISTRIBUTED LOAD
           SIMPLE SPAN, LATERALLY SUPPORTED
                WORKING STRESS DESIGN
***********************************************************************
                                        C-CHANNEL-(mm)
                                      25
  THICKNESS      = 2.5 mm            ***            ***
  IR             = 5   mm            *                *  90
                                     ****************
  Fy= 345 MPa                               250
  Fu= 450 MPa
***********************************************************************
```

Span-L (mm)	Defl'n (mm)	Load (kN)	UDL to Deflection Limits- (kN) L/180	L/240	L/360
1000	1.96	37.4	37.4	37.4	37.4
1500	4.42	24.9	24.9	24.9	23.5
2000	7.85	18.7	18.7	18.7	13.2
2500	12.27	14.9	14.9	12.7	8.5
3000	17.66	12.5	11.7	8.8	5.9
3500	24.04	10.7	8.6	6.5	4.3
4000	31.40	9.3	6.6	5.0	3.3
4500	39.75	8.3	5.2	3.9	2.6
5000	49.07	7.5	4.2	3.2	2.1
5500	59.37	6.8	3.5	2.6	1.7
6000	70.66	6.2	2.9	2.2	1.5
6500	82.93	5.7	2.5	1.9	1.3
7000	96.18	5.3	2.2	1.6	1.1
7500	110.41	5.0	1.9	1.4	.9
8000	125.62	4.7	1.7	1.2	.8
8500	141.81	4.4	1.5	1.1	.7
9000	158.98	4.2	1.3	1.0	.7
9500	177.14	3.9	1.2	.9	.6
10000	196.28	3.7	1.1	.8	.5

```
***********************************************************************

     SECTION PROPERTIES AND DESIGN DATA
     **********************************

       Iy  (mm4)        1220.80 x 1000
       Sy  (mm3)          19.06 x 1000
       M   (Nm)         4669.57
       V   (kN)           51.75
       Pint  (kN)           19.1 -applies only for a
       Pext  (kN)            9.8 -bearing length of   85.0 mm

***********************************************************************
```

TABLE 5

****

METAL SHAPES INCORPORATED

*** BEAM LOAD TABLES -METRIC ***

TOTAL ALLOWABLE UNIFORMLY DISTRIBUTED LOAD
SIMPLE SPAN, LATERALLY SUPPORTED
WORKING STRESS DESIGN

****

C- Channel-(mm)
250

THICKNESS = 2.5 mm
IR = 5 mm

90

25

Fy= 345 MPa
Fu= 450 MPa

Span-L (mm)	Defl'n (mm)	Load (kN)	UDL to Deflection Limits- (kN) L/180	L/240	L/360
1000	2.10	34.7	34.7	34.7	34.7
1500	4.72	23.1	23.1	23.1	20.4
2000	8.39	17.4	17.4	17.2	11.5
2500	13.10	13.9	13.9	11.0	7.4
3000	18.87	11.6	10.2	7.7	5.1
3500	25.68	9.9	7.5	5.6	3.8
4000	33.55	8.7	5.7	4.3	2.9
4500	42.46	7.7	4.5	3.4	2.3
5000	52.42	6.9	3.7	2.8	1.8
5500	63.42	6.3	3.0	2.3	1.5
6000	75.48	5.8	2.6	1.9	1.3
6500	88.58	5.3	2.2	1.6	1.1
7000	102.74	5.0	1.9	1.4	.9
7500	117.94	4.6	1.6	1.2	.8
8000	134.19	4.3	1.4	1.1	.7
8500	151.48	4.1	1.3	1.0	.6
9000	169.83	3.9	1.1	.9	.6
9500	189.23	3.7	1.0	.8	.5
10000	209.67	3.5	.9	.7	.5

****

SECTION PROPERTIES AND DESIGN DATA

****

Iy eff (mm4)	984.27	x 1000
Iy defl (mm4)	1062.18	x 1000
Sy eff (mm3)	17.67	x 1000
M (Nm)	4340.07	
V (kN)	51.75	

****

TABLE 6

```
******************************************************************

        METAL  SHAPES  INCORPORATED

          *** COLUMN LOAD TABLES - METRIC ***

             ALLOWABLE AXIAL LOADS
             WORKING STRESS DESIGN
******************************************************************
C-Channel- (mm)
    90
   ****                   Thickness    = 2.5 mm
   *  * 25                IR           = 5 mm
250*
   *  *                   Fy = 345 MPa
   ****                   Fu = 450 MPa

******************************************************************
```

Effective Length (KL) in respect to least Radius of Gyration (mm)	Load (kN)	Local Buckling Factor Q=Qf*Qa
1000	166.0	.711
1500	155.3	.720
2000	140.6	.735
2500	121.8	.757
3000	98.8	.794
3500	78.4	.841
4000	64.6	.888
4500	54.8	.933
5000	47.6	.976
5500	41.3	1.000
6000	35.2	1.000
6500	30.0	1.000
7000	25.9	1.000
7500	22.6	1.000
8000	19.8	1.000
8500	17.6	1.000
9000	15.7	1.000
9500	14.1	1.000
10000	12.7	1.000

****

SECTION PROPERTIES AND DESIGN DATA
****

A	(mm2)	1148.18
Ix	(mm4)	10934.02 x 1000
Iy	(mm4)	1220.80 x 1000
Rx	(mm)	97.59
Ry	(mm)	32.61
Xo	(mm)	65.88
J	(mm4)	2392.03
Cw	(mm6)	1.573 x 10^10

****

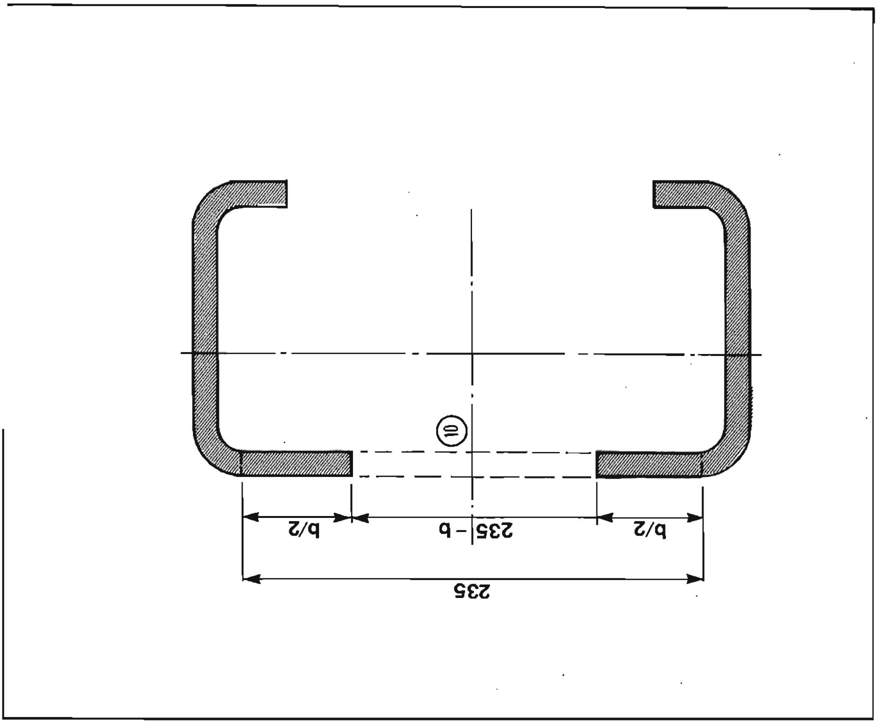

Figure 2

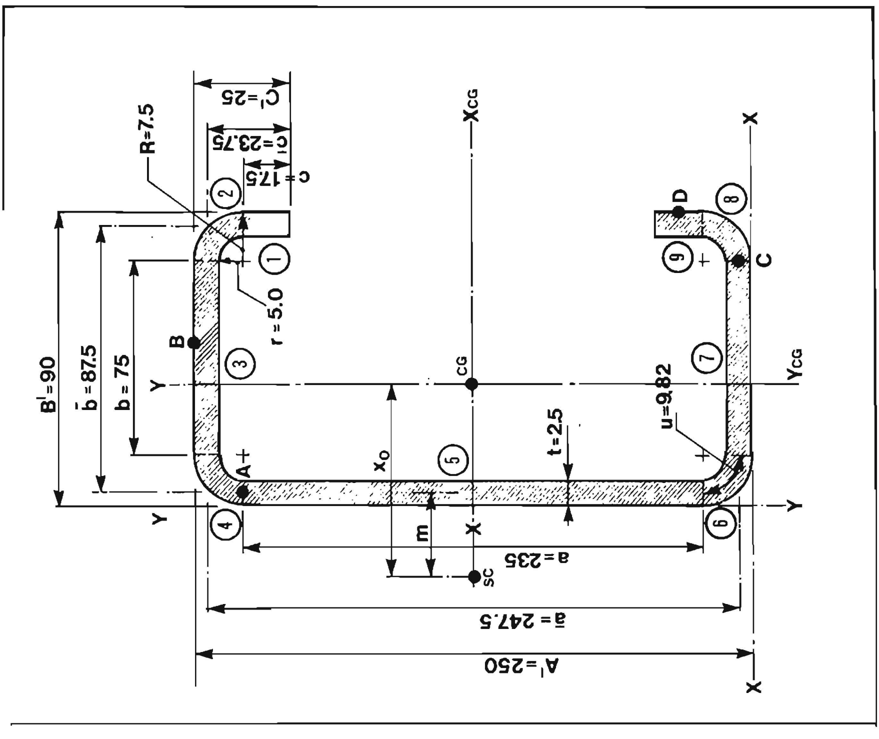

Figure 1

SUMMARY

DESIGN AIDS FOR STIFFENED CHANNELS

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Perhaps the most widely used cold formed steel section is the stiffened channel. While the shape of this section is simple, the calculation of its section properties and its beam and column strengths is deceptively complex. Published design standards for cold formed members are invaluable when undertaking engineering calculations for stiffened channels. However, a considerable number of assumptions must be made by the engineer when using such standards. It is for this reason that the properties and strengths of the exact same stiffened channel differ from one handbook to another and from one manufacturer's literature to another's.

This paper sets forth precise methods for calculating the section properties and the beam and column strengths of cold formed stiffened channels. Example calculations, based on Canadian Standards Association Standard S136 "Cold Formed Steel Structural Members", are given. Example beam and column tables, computer-generated, are also included. The authors' wish is that this paper will lead to the standardization of engineering calculations for cold formed stiffened channels. Their ultimate hope is that North American handbooks and design aids will provide consistent properties and strengths for cold formed channels.